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Simulation of the John Day Dam North Fish Ladder Entrance and Auxiliary Water Supply Facilities

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Abstract

In the early 1960's, the John Day Dam North Fish Ladder was constructed between the navigation-lock and the overflow spillway. The ladder system provides a bypass route for upstream migrating fish (salmon and shad) on the north side of the Columbia River. High volumes of flow (800 -1600 cfs) are used to attract the fish from the tailrace into the ladder entrance, consisting of two submerged telescoping weirs. Once in, the fish use the ladder overflow section to scale the 100-foot head rise to the Forebay exit. Most of the entrance attraction water is supplied by an auxiliary water system (AWS), fed by large pumps that draw from the tailrace lock approach channel. Since construction, the hydraulic criteria have become more demanding and higher attraction flows are required. The Portland District was tasked to determine the reason why the system fails to meet these new criteria. A numerical hydraulic model was developed to aid in this evaluation study.

The numerical model provides a one-dimensional flow simulation of the spatially varied flow conditions in the prototype. The traditional laws of continuity, linear momentum and energy are used to mathematically simulate each element of the lower ladder system. This includes entrance weirs; transportation channel; diffusers that distribute the attraction flow throughout the lower ladder; and the AWS pumps. The model has been developed using the VISUAL BASIC tools of the Microsoft EXCEL spreadsheet and was successfully calibrated with hydraulic field data. The model was also used to explore potential corrective improvements.

Introduction

There are two adult bypass fishway systems at John Day dam, each located on opposite sides of the Columbia River. These fishway systems are vital for the successful upstream migration of salmon to the upper reaches of the Columbia River and tributaries. The John Day North Fishway receives and passes fish approaching from the Washington shore of the river. This fishway does not have the capacity to be operated under current adult fish passage criteria. The Portland District (CENWP) was tasked with conducting an evaluation study of the system. In the process, a numerical model was developed as an evaluation tool. This paper includes a description of the numerical model and a summary of evaluation results using the model.

System Description

The John Day North Fishway is located between the Navigation Lock and the Spillway on the Washington side of the Columbia River [3, 4]. The complete fishway consists of three main components. The first is the entrance and the collection channel; the second is the overflow ladder weir section; and the third is the exit and ladder control section. The following figure displays the lower portion of the system, which is the focus of this paper.

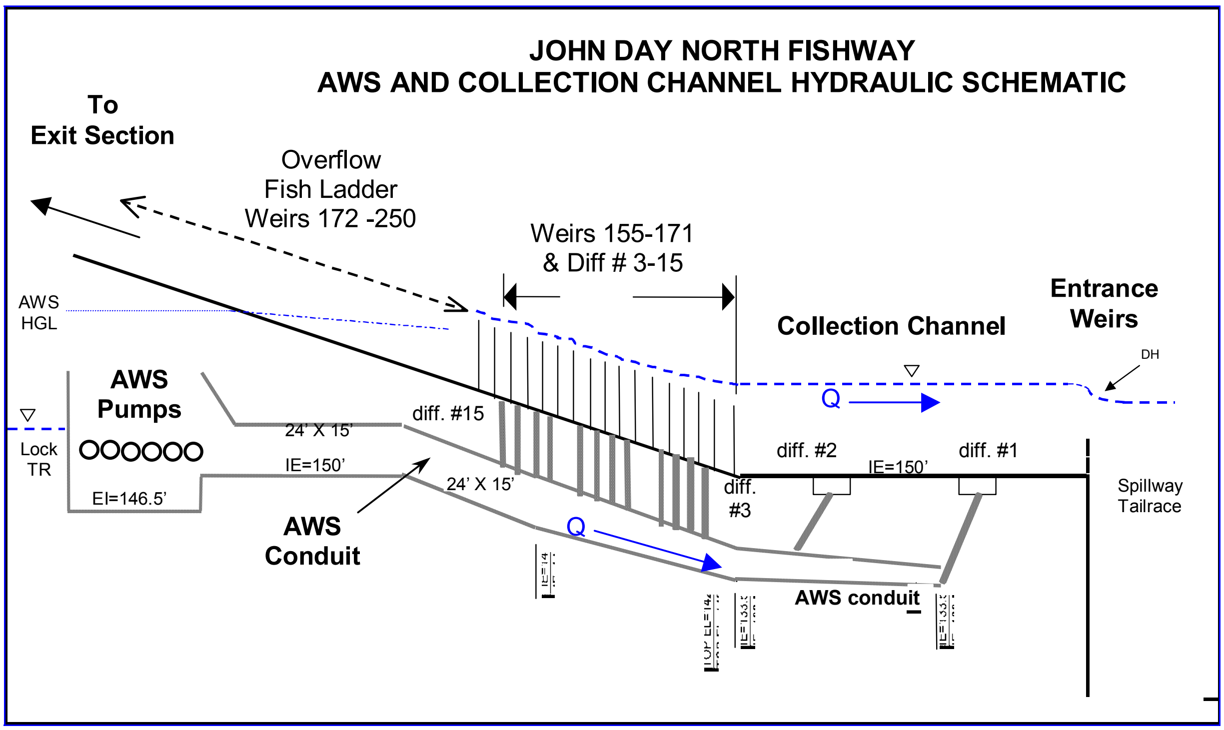

Figure 1 Hydraulic Schematic of John Day North Fishway: Collection Channel and Auxiliary Water System

Entrance. The entrance is the location where the upstream migrating fish enter the fishway. The large volumes of flow through the entrance are used to lure the fish into the system from the tailrace. The entrance consists of two adjacent 12-foot wide telescoping submerged weirs. The adult fisheries criteria for the entrance operations are 8 feet of weir submergence (Tailwater level – weir crest elevation) and 1-2 feet of water surface differential head across these weirs. The minimum weir crest level is

150.2 feet, just inches above the channel invert. Consequently, the project tailrace level is normally kept above 158 feet to maintain the weir submerge criteria.

Figure 2 A. Fishway Entrance; B. Overflow Ladder Weirs

Collection Channel and Overflow Ladder Weirs. As the fish move into the system, the collection channel connects the entrance to the overflow ladder weirs. Channel velocities between 1.5 and 4 ft/s must be maintained to keep the fish moving forward. The overflow ladder weirs are a series of cascading weirs that allow the fish to ascend the 100–foot head rise over John Day dam. The weir crests step up in one-foot increments along a 1:16 channel slope. At the base of each weir, there are two 18-inch square orifices. Most salmon use the orifices--which are most effective when the head differential is 1. 0 – 1.3 feet across the ladder weirs. As the fish near the top, they are guided through a counting station and exit section to the forebay. The ladder head control system provides constant ladder head and flow from the top regardless of forebay level. This discharge rate is 75-90 cfs (depending on the ladder head), only about 10% of the flow that discharges from the entrance.

Figure 3 A. AWS Pump Motor; B. Pump Building, Intakes and Discharge Channel

Auxiliary Water Supply (AWS) Pumps. The bulk of the entrance flow is delivered by the auxiliary water supply (AWS) system. Auxiliary water is fish-free water that is supplied by six low head, 73 inch, high volume pumps with a rated capacity of 300 cfs at 3 ft. of head. These pumps draw from the tailrace lock approach channel and raise enough head to circulate the flow through the AWS and entrance back to the tailrace in the spillway side. The pumps were originally manufactured by the Pelton Division of Baldwin Lima Hamilton. The motors are 157 horsepower, and were originally manufactured by Westinghouse and rated at 1783 RPM. This rotational speed is geared down to 116 RPM by a gear reduction unit. The full load amperage of these motors is 19.1 amps and the maximum allowable is 22.1 amps. The pumps and motors have been in service for about 40 years. Other than normal maintenance, they remain in good condition and maintain original design flow capacity.

AWS Conduit and Diffusers. A large rectangular conduit transports the AWS pump flow into the fishway channels via floor diffusers. Diffusers are large openings in the fishway channel floor through which AWS water slowly rises (velocity < 1 ft/s) into the channel. The diffusers are covered with grating to prevent adult fish entry into the AWS. The diffuser system consists of two large diffusers in the floor of the collection channel and 13 smaller diffusers in the lower pools of the ladder system. The number of small diffusers (3-15) that function at any one time is completely dependent on the elevation of the tailwater. Each of these diffusers has a "stovepipe" weir—built 3 feet higher than the adjacent ladder weir--that prevents or limits AWS discharge into the diffuser depending on the AWS HGL level. The design intent was to allow a passive increase of collection channel flow as the tailwater level rose so that channel velocities could be maintained at acceptable levels.

Control System. The original design provided a control system, but it was never satisfactory. The pumps and weirs have since been operated in manual control. The pumps do have a sophisticated overload protection logic control system that activates a shutdown procedure when a pump motor is in danger of overheating. Data for the water level elevations in the tailrace, collection channel and weir crests are obtained from stillwells and converted to digital information for continuous monitoring.

Historical Background

The John Day Dam unit of the dams on the Lower Columbia River was designed and constructed in the 1950's and 1960's [3], [4]. The system was designed under a different set of adult fisheries criteria, which emphasized channel velocities. The system could be operated satisfactorily to meet the original design criteria.

The adult fisheries bypass criteria were eventually changed and the focus shifted to increased attraction flow through the entrances. In order to meet the new criteria, the project operators had to run more pumps at the lower tailrace levels. The limitations of the system soon became apparent and it was recognized that only two to three pumps could be operated simultaneously for any length of time. A fourth pump could be turned on, but shortly thereafter the most upstream pump would experience an overload on the motor and automatically shut itself off. The system deficiency persisted in spite of improvements made to the electrical capability of the system.

Consequently, the entrance weirs could not be operated within current criteria. When the project tried to operate both weirs at the required 8-feet submergence, the entrance heads were too low. The head criterion could only be met only if the weirs were raised above minimum allowable submergence levels. The project ultimately settled on a single weir operation that meets both the submergence and head criteria. However, this situation was not satisfactory to regional fish agencies (National Marine Fisheries Service, et. al.) and an evaluation study was requested.

The Portland District (CENWP) of the Corps of Engineers was tasked to determine the cause of the operational deficiency and recommend alternative solutions towards meeting the goal of operating two entrance weirs in criteria. This study is documented in the *John Day North Fish Ladder Evaluation Study Report*, completed in October, 2002 [7]. In the process, the Portland District Hydraulic Design Section (EC-HD) developed a numerical model of the fishway to perform the evaluation and develop recommendations for potential solutions.

Evaluation Procedures

Prototype data. During the summer of 2001, a team of engineers and technicians conducted multiple and single prototype pump tests to describe the collection channel operations under a range of flows [5], [7]. On each test the different hydraulic boundary conditions, pressures and heads were recorded, as well as the electrical parameters [2]. The tests were performed at night to avoid interrupting fish passage, which mainly occurs during daylight hours. During the tests, the tailrace was held constant (± 0.5 feet) through coordination with the John Day project and Northwest Division Reservoir Control Center. Flow measurements for the total AWS discharge were measured through an air vent using an acoustic Doppler velocimeter (adv). The ladder flow from the exit control section was estimated and added to the AWS flow to determine the total discharge through the fishway entrance. The test results generally confirmed the manufacturer's pump curves and provided valuable data for entrance head versus flow rate. The prototype data are summarized in Table 1 below.

After the prototype data was assimilated, work was started on a one dimensional hydraulic simulation model.

Model Development. The model was developed as an integral part of the *John Day North Fish Ladder Evaluation Study Report (October 2002)* [7]. The testing and documentation of the model are described in the report. The first objective of the model was to provide accurate simulations of the existing system performance for evaluation with respect to criteria at the full range of project conditions. A second objective was to provide a tool for the designers that would allow maximum flexibility in modifying the model to consider potential system modifications. A third objective of was providing project operators and biologists the means to test proposed changes to their normal operations. The code is accessible for future engineers to make modifications and geometric alterations to evaluate future design or operational problems. Most of the previous models developed in EC-HD utilized FORTRAN, but the recent engineering graduates are more proficient with spreadsheets and VISUAL BASIC APPLICATIONS (VBA). Since it was anticipated that the extensive library of FORTRAN subroutines would not be needed for this application, VBA was selected as the language to develop this model.

Table 1: John Day North Fishway Prototype Test Data Summary

Multiple Pump Data									
TEST NUMBER	Number of Pumps	AWS Flow Rate (cfs)	Total Entrance Flow Rate (cfs)	Number of Open Entrance Weirs	Tailwater Elev (ft)	Weir Elev. (ft) (average if 2)	Weir Head (ft)	Weir Submerg-ence (ft)	WS Gradient Across AWS System (ft)
2P34L	2	623	701	1	159.6	150.3	0.96	9.3	1.40
2P34M	2	596	674	1	159.4	151.5	1.57	7.9	1.08
2P34H	2	589	667	1	159.5	152.8	2.09	6.6	0.97
3P134M-2W	3	888	966	2	159.0	151.5	0.67	7.5	2.49
3P134L-2W	3	886	964	2	158.9	150.4	0.41	8.5	2.75
3P134H-1W	3	864	942	1	160.0	150.3	1.57	9.7	1.66
3P134H-2W	3	861	939	2	159.6	153.9	1.32	5.7	2.04
4P1345H-2W	4	1097	1175	2	159.4	150.5	0.42	8.9	3.49
Single Pump Data									
Test No.	No. Pumps	AWS Q	Entrance Q	Open Entrances	TW	Weir Elev	Weir Head	Submerg-ence	AWS Gradient
P1L	1	335	413	1	159.4	153.8	1.33	5.6	0.12
P1M	1	310	388	1	159.6	155.7	1.99	3.9	0.19
P1H	1	293	371	1	159.6	157.3	2.83	2.3	0.08
P3L	1	322	400	1	159.4	153.8	1.31	5.6	0.53
P3M	1	311	389	1	159.4	155.7	2.13	3.7	0.46
P3H	1	281	359	1	159.7	157.9	3.09	1.8	0.20
P3HH	1	232	310	1	159.2	159.6	4.49	-0.4	0.05
P4M	1	320	398	1	159.8	155.8	1.97	4.0	0.35
P5M	1	318	396	1	159.7	155.8	2.01	3.9	0.33

Description of the Model

Input Data. The model requires the following input data:

- Tailwater elevations at the fishway entrance and the Lock channel
- Entrance weir crest elevations
- Initial guess for entrance head (water level drop across entrance)
- Number of pumps in operation.
- Upper ladder flow rate: either give flow rate or give upper ladder head and compute upper ladder flow rate.

Computational Steps. The following steps represent the computational procedure taken by the program to simulate the system:

1. Using the assumed entrance head (Δh) compute the fishway discharge using the weir equation and Villomente correction for submergence.
2. Using energy principles, compute the collection channel water surface, or hydraulic gradeline (HGL), back to diffuser #1.
3. Assume an HGL in the AWS conduit at diffuser #1.
4. Compute the diffuser #1 discharge from the difference in HGL's.
5. AWS Q = Diffuser #1 Q; Fishway Q = Fishway Q – Diffuser #1 Q

6. Using linear momentum principles, compute water surface upstream of diffuser #1.
7. Return to energy principles to extend the HGL's for the collection channel and AWS back to diffuser #2.
8. Using similar logic, compute diffuser #2 discharge and upstream water depth at base of weir #155;
9. AWS Q = AWS Q + diffuser 2 Q, Fishway Q = Fishway Q – diffuser # 2 Q
10. Using Fishway Q, compute the change in head over weir 155.
11. Starting with diffuser #3: If AWS HGL > than the stove-pipe weir crest, then compute the diffuser Q as function of weir crest and the differences in HGl's.
12. AWS Q = AWS Q + diffuser Q; Fishway Q = Fishway Q – diffuser Q.
13. Compute head loss over next weir and repeat steps 11-12 through diffuser 4-15 and weirs 156-171; Note fishway Q above weir 171.
14. Determine flow regime in AWS base on level of HGL above diffuser 13.
15. Project the HGL back to the pump discharge channel.
16. Starting with downstream operating pump: compute the pump discharge and HGL upstream of pump using linear momentum principles; repeat steps for all remaining operating pumps and complete the backwater profile in the discharge channel.
17. Check pump imbalance (difference between total AWS and total pump flow): if difference is less than specified error tolerance, go to step 18; else modify the AWS HGL and iterate on system again (return to step 3).
18. If the pump imbalance is within the tolerance, then check imbalance between set ladder flow and Fishway Q above weir 171: if difference is within set tolerance, then DONE; otherwise modify the entrance head to compute the system discharge and correct the ladder flow (return to step 1).

The numerical difficulty was to solve system continuity: the total AWS flow should equal the sum of the pump discharges; and the difference between the total entrance flow and total AWS flow would equal the upper ladder flow. The presence of diffusers 3-15 often introduced a stepwise function in the computations, as one of these diffusers would often come on or drop out of the analyses depending on the readjustment of the AWS HGL or the entrance head. A combination of numerical schemes was tried with each having both strong and weak points. The SECANT [2] method was tested and in some cases worked very well, but on occasion it seemed to have difficulty when the function was relatively flat. The other method that had some success was the BISECTION [2] method. This method is numerically inefficient and mathematically inelegant, but it is more stable. The SECANT method normally converges faster, but it can experience instability working past a stepwise function--sometimes leading to erratic and divergent adjustments to the independent variables. The present program logic uses a combination of these two methodologies to solve total system continuity.

Model Output. The model output provides the following data:

- Entrance weir head and total flow over entrances
- Flow over each entrance weir

- Fishway channel velocities
- Flow rates and average velocities through each diffuser
- Water levels in Fishway (in graph form)
- Hydraulic gradelines in AWS and pump discharge channel
- Discharge rates and head for each pump

Description of Numerical and Hydraulic Methodologies in Model

Entrance Flow Rates. The discharge through the entrance weir(s) is computed using the standard weir equation (See Equation 1) with certain coefficients and adjustments. The first parameter that must de determined is the weir discharge coefficient (Cw); which is a function of the ratio of the weir height (P) above the invert and the head (H) over the weir crest in the upstream approach channel. (H = TW level + Δh entrance – weir crest elevation) This relationship for Cw = f(H/P) was empirically determined from the prototype measurements. While the magnitude of the empirical Cw curve was lower than theoretical Cw curve, the shapes were similar. Using the weir equation, the 'free' discharge is computed. The Villemonte correction for weir submergence is applied to compute the adjusted weir flow.

Collection Channel Backwater Profile. The water level just upstream of the entrance weirs represents the starting downstream water surface level for the collection channel. Given this and the total entrance discharge, the backwater profile is computed using the manning equation (See Equation 2) back to diffuser 1. After the diffuser discharge is computed, water surface upstream of diffuser 1 is computed using the momentum principle (Equation 4). The diffuser flow is deducted from the total fishway flow for the backwater computations upstream of the diffuser. This process is repeated up to and past diffuser 2 and to weir 155.

Overflow Ladder Weirs. Starting at weir 155, the downstream water level and fishway flow rate is known. The head loss over the weir is computed using Equation 1. Cw is a constant for the ladder weirs. If there is a diffuser in the ladder pool, the computed diffuser Q is deducted from the fishway flow for the next upstream weir. This process is repeated for weirs 156-171 until a final fishway Q above weir 171 is obtained. This flow rate is ultimately compared with the input upper ladder flow to determine is system continuity is met (Equation 3).

Large Diffuser Flow Calculations. The simulation of the large diffusers that discharge into the floor of the collection channel was a challenge. The prototype has numerous ports that supply the diffuser geometry just below the floor. The head loss in this system of ports and risers and the diffusers is simulated with the principle of "equivalent length of pipes". Data was available from the prototype tests and the head loss in the system was determined for these boundary conditions. Diameters and pipe lengths were determined that would produce an equal amount of head loss to simulate this portion of the system. Then in order to compute the flow added to the collection channel from these large diffusers, the program assumes a p/γ value for the first diffuser this allows the flow added the large diffusers to be calculated. This procedure is similar to the assumption of a p/γ at the junction in the typical academic "three reservoir" hydraulics problem. This parameter is adjusted each iteration until the difference between the total AWS discharge and the pump discharge (pump imbalance) is within the selected tolerance.

AWS Conduit HGL Backwater. The HGL in the AWS is computed from diffuser 1 and above using the Manning equation (Equation 3). The HGL and discharge at diffuser 1 are the starting points for the AWS computations. After passing the next diffuser, the computed diffuser flow is added to the AWS flow for the remaining upstream computations. This process is repeated past diffuser 15, then a check must be made to determine if the AWS flow is pressurized or open channel upstream.

Small Diffuser Flow Calculations. At each small diffuser (3-15), the level of the AWS HGL is compared to the elevation of the stovepipe weir for the diffuser. If the AWS HGL is less the stovepipe weir crest, then diffuser Q = 0; otherwise the diffuser Q is solved using equation 1. In this equation, the water level in the ladder pool of this diffuser represents the downstream water level on the diffuser stovepipe weir. If this is also higher than the diffuser weir crest, then the Villomente correction must be applied.

Flow Regime at AWS Grade Break. There is grade change in the AWS conduit above diffuser 15: on the downstream side, the conduit slopes downward; upstream, the conduit is level. During the prototype testing, it was observed at the grade break that the conduit was operating under atmospheric conditions for tailwater elevations less than $163\pm$. This tailwater level is above the annual median 161.7 feet and no prototype measurements were taken under such high levels. This made the hydraulic simulation of the AWS very challenging. The program logic for the AWS starts by computing the flows in the large diffusers in the floor of the collection channel. The computations then proceed back up the AWS through each of the small diffusers locating the HGL with respect to the elevation of the control weir in each small diffuser. Upon reaching diffuser #15, the HGL is projected back to the section where the slope of the AWS changes from a horizontal to a gradually varied flow (GVF) steep slope. If the projected HGL is above the soffit of the horizontal portion of the AWS, the next losses in this section are computed as pressure flow. If the projected HGL is below the soffit, the water surface is computed as open channel flow. In either case, Equation 2 is applied.

Pump Discharge and Discharge Channel Water Surface. After computing the HGL back to the pump discharge chamber, the average head across each pump is computed to determine its discharge. The pump head is the difference between the average head calculated in the discharge chamber and the tailwater in the lock channel. The program computes this water surface change for each pump discharging into the channel. This procedure of defining the slope of the water surface in the pump discharge channel uses the spatially varied flow and momentum principles (Equation 4). This has been identified as one of the sources of the pump shut-off problem—as the most upstream pump, where the water level and pump head is greatest, always shuts off first.

In computing the discharge for each pump, the manufacturer's head-discharge curve was regressed allowing the discharge to be computed as a function of the differential head for each pump. The cubic equation has four coefficients that are displayed on the spreadsheet. Any future modification to the pump impellers or any other pump parameters (i.e. RPM, diameter, etc.) can be described by this equation with a corresponding set of coefficients describing any new H-Q relationship. The manufacturer's H-Q curve is very flat for this type of units and 6-12 inches of head difference between the first operating pump and the last operating pump will mean a difference of approximately 40-50 cfs. The amperage drawn by each pump motor is calculated both theoretically and the also by regressing the amperage/head data taken during the prototype testing. The program computes both these parameters, and the cell background color turns red when exceeding the maximum allowable amperage (22.1) of the motors. This feature also allows the user to modify these calculations for any modification to the motors that would change their max allowable amperage.

Equations

Submerged Weirs. There are at least three instances where the crests of weirs are below the down stream water surfaces: (1) entrance weirs; (2) small diffuser stovepipe weirs; and (3) submerged lower ladder weirs (#155-#171). If the down stream water surface is above the crest of the weir, the Villemonte [1] coefficient is computed and applied to the weir discharge equation. The other coefficient affecting the efficiency of the weir is the weir coefficient (Cw). This has been studied extensively by CENWP-EC-HD and a full discussion can be found in [6, 7].

$$Q = LC_w 2/3\sqrt{2g}H^{1.5}\left[1-\left(\frac{H_2}{H_1}\right)^{1.5}\right]^{0.385} \quad (1)$$

Q = Weir Discharge
L = Length of weir
C_w = Weir Coefficient
H_1 = Height of Energy Grade Line up stream of weir crest
H_2 = Height of Energy Grade Line down stream of weir crest
g = Acceleration of gravity (32.2 ft/s)

Friction Losses in Open Channel and Pressurized Flow Regimes. Manning equation was used exclusively due to the rectangular geometry. The computation of the frictional loss in a reach of the system is:

$$h_l = \left[\frac{Q*n}{1.486*A*R^{\frac{2}{3}}}\right]^2 * L \quad (2)$$

h_l = Frictional head loss in specific reach of the system
Q = Channel Discharge
n = Manning friction coefficient
L = Length of reach
A = Cross section of the average area
R = Hydraulic Radius (A/WP)

Continuity [8]. Continuity represents the mass balance of the system flows.

$$A_1V_1 = A_2V_2 \quad (3)$$

Linear Momentum. The water surface change across a channel section with inflow (such as a diffuser) is solved using the linear momentum principle. A sum of the pressures forces and momentum flux is taken about a control volume (cv) representing the channel section. The equation below is used when transverse flow enters the channel from a direction perpendicular to the velocity in the channel. The expression may then be reduced to geometry and flows and solved algebraically. about a Control Volume with Inflow (steady state) [8]

$$F_1 - F_2 = M_2 - M_1 \quad (4)$$

F_1 = Upstream pressure force on Control Volume (cv) = $\gamma * P_1 * A_1$
F_2 = Downstream pressure force on cv = $\gamma * P_2 * A_2$
γ = Unit weight of water = 62.4 lbs/ft^3
P = Average pressure against upstream or downstream cv surface
A = Area of upstream or downstream cv surface
M_1 = Momentum flux from the upstream face of cv = $\rho * Q * V_1$
M_2 = Momentum flux from the downstream face of cv = $\rho * Q * V_2$

Comparison of Model Simulations with Prototype data

The Manning n values were calibrated from the highest flow test (4 pumps). The comparative results showed that the model was reliable as a tool for the evaluation study of the fishway. The comparison of model and prototype data under equivalent boundary conditions is shown in the following table:

Table 2 Comparison of Model Simulations and Prototype Data

MULTIPLE PUMP TESTS

BOUNDARY CONDITIONS **TRANSPORTATION CHANNEL and AWS**

	Tailwater Elevation	Lock channel Elevation	Delta H	Elevation Weir 1	Elevation Weir 2	WS u/s Elevation	WS between Diff. 1 & 2	WS u/s Diff. 2	HGL El @ Diff. 3	HGL El @ Diff. 15	WS El d/s end DC	HGL El u/s end DC	Pump Discharge
Multiple Pump Test ---- Pumps #1,#3, and #4(2 Weirs) ---- Low Head													
TEST DATA	158.94	159.12	0.41	150.50	150.30	159.35	159.80	160.20	160.99	161.37	161.94	162.10	886
SIMUL. DATA	158.94	159.12	0.35	150.50	150.30	159.41	159.76	159.97	161.13	161.38	161.75	162.09	895
difference	**0.00**	**0.00**	**0.06**	**0.00**	**0.00**	**-0.06**	**0.04**	**0.23**	**-0.14**	**-0.01**	**0.19**	**0.01**	**-9**
Multiple Pump Test ---- Pumps #1, #3, and #4(2 Weirs) ----Medium Head													
TEST DATA	158.96	159.14	0.67	151.60	151.40	159.63	160.20	160.40	161.18	161.50	161.96	162.03	889
SIMUL. DATA	158.96	159.14	0.79	151.60	151.40	159.85	160.16	160.35	161.40	161.64	162.02	162.34	880
difference	**0.00**	**0.00**	**-0.12**	**0.00**	**0.00**	**-0.22**	**0.04**	**0.05**	**-0.22**	**-0.14**	**-0.06**	**-0.31**	**9**
Multiple Pump Test ---- Pumps#3, and #4(1Weir) ---- High Head													
TEST DATA	159.45	159.59	2.09	166.30	152.80	161.54	161.70	161.60	161.92	162.01	162.26	162.50	589
SIMUL. DATA	159.45	159.59	2.12	166.30	152.80	161.60	161.70	161.77	162.13	162.23	162.63	162.76	585
difference	**0.00**	**0.00**	**-0.03**	**0.00**	**0.00**	**-0.06**	**0.00**	**-0.17**	**-0.21**	**-0.22**	**-0.37**	**-0.26**	**4**
Multiple Pump Test ---- Pumps #3, and #4(1 Weir) ---- Medium Head													
TEST DATA	159.36	159.22	1.57	166.30	151.50	160.93	161.10	161.10	161.55	161.58	161.74	162.01	596
SIMUL. DATA	159.36	159.22	1.32	166.30	151.50	160.72	160.85	160.93	161.40	161.51	161.90	162.05	600
difference	**0.00**	**0.00**	**0.25**	**0.00**	**0.00**	**0.21**	**0.25**	**0.17**	**0.15**	**0.07**	**-0.16**	**-0.04**	**-4**
Multiple Pump Test ----Pumps #3, and #4(1 Weir) ----Low Head													
TEST DATA	159.59	159.71	0.96	166.30	150.26	160.55	160.80	160.80	161.20	161.29	161.87	161.95	623
SIMUL. DATA	159.59	159.71	0.98	166.30	150.26	160.61	160.75	160.84	161.36	161.48	161.87	162.03	618
difference	**0.00**	**0.00**	**-0.02**	**0.00**	**0.00**	**-0.06**	**0.05**	**-0.04**	**-0.16**	**-0.19**	**0.00**	**-0.08**	**5**
Multiple Pump Test ---- Pumps #1,#3, and #4(1 Weir) ---- High Head													
TEST DATA	159.96	159.97	1.57	166.30	150.26	161.53	161.80	161.90	162.65	162.72	163.08	163.19	864
SIMUL. DATA	159.96	159.97	1.60	166.30	150.26	161.63	161.82	161.95	162.66	162.88	163.28	163.55	850
difference	**0.00**	**0.00**	**-0.03**	**0.00**	**0.00**	**-0.10**	**-0.02**	**-0.05**	**-0.01**	**-0.16**	**-0.20**	**-0.36**	**14**
Multiple Pump Tests ---- Pumps #1, #3, and #4(2 Weirs) ----High Head													
TEST DATA	159.96	159.97	1.32	154.00	153.80	161.53	161.40	161.40	162.19	162.54	162.82	162.98	864
SIMUL. DATA	159.96	159.97	1.46	154.00	153.80	161.49	161.68	161.82	162.55	162.77	163.18	163.44	859
difference	**0.00**	**0.00**	**-0.14**	**0.00**	**0.00**	**0.04**	**-0.28**	**-0.42**	**-0.36**	**-0.23**	**-0.36**	**-0.46**	**5**
Multiple Pump Test ---- Pumps #1, #3, #4, and #5(2 Weirs) ---- High Head													
TEST DATA	159.36	159.57	0.42	150.60	150.40	159.78	160.60	160.80	162.01	162.70	163.10	163.27	1097
SIMUL. DATA	159.36	159.57	0.43	150.60	150.40	159.95	160.40	160.67	162.10	162.48	162.88	163.34	1116
difference	**0.00**	**0.00**	**-0.01**	**0.00**	**0.00**	**-0.17**	**0.20**	**0.13**	**-0.09**	**0.22**	**0.22**	**-0.07**	**-19**

Summary of Fishway Problem

The existing system was designed under different criteria and cannot provide sufficient entrance flows to meet the current adult fish passage criteria. Due the configuration of the entrances, the current criteria implicitly require maximum entrance flow rates at minimum tailrace levels. Here the crest of the entrance weirs barely project above the channel floor and create minimal hydraulic constriction needed to induce the required head drop across the entrance. At the same time, the AWS system hydraulic conveyance capacity is reduced to a minimum as a function of tailrace. Higher heads are imposed on the pumps from the combination of higher entrance head requirements and reduced AWS conveyance. Thus at low tailrace levels, the AWS pumps are expected to produce more flow against higher head. The existing pump system has a narrow range of head and flow capacities and cannot be operated meet these flow requirements except at very high tailrace levels. The following figure illustrates the differences between the flow curves as required by the current criteria, original criteria and system capacity.

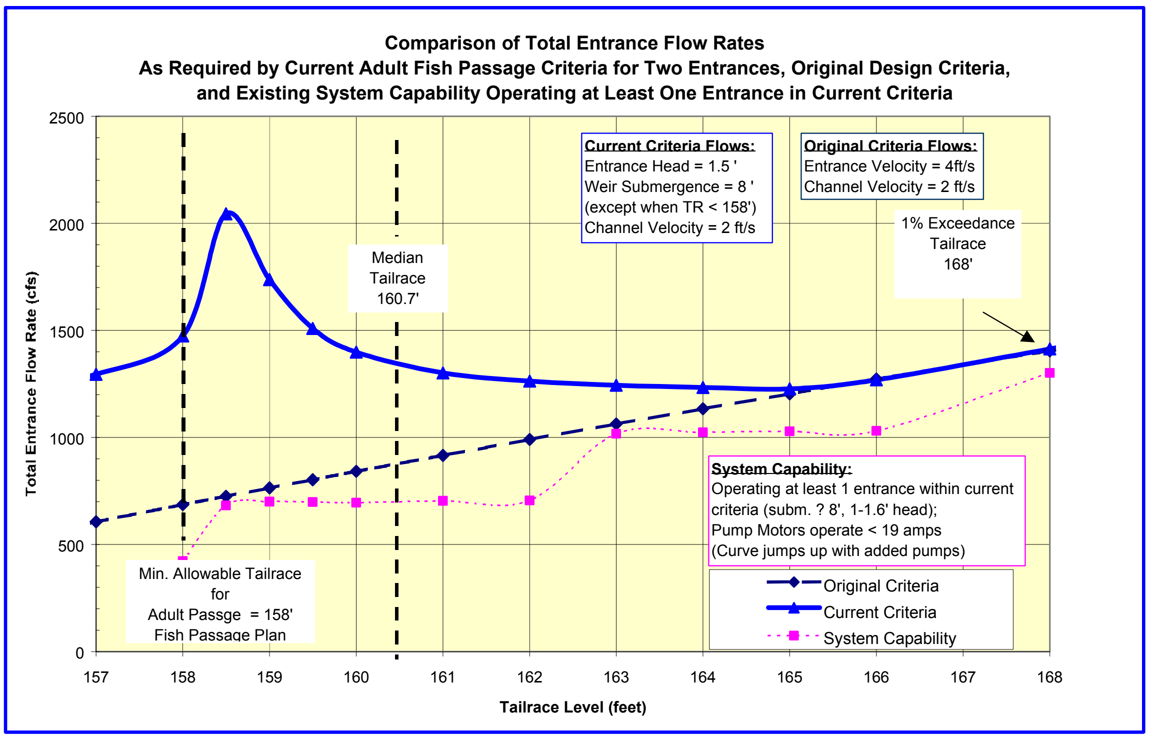

Figure 4 Entrance Flow Requirements as Required by Current Adult Fish Passage Criteria and Original Design Criteria

Potential Alternatives for Fishway Upgrades

The *John Day North Fish Ladder Evaluation Study Report* [7] contains several alternatives that will either fully meet the criteria or incrementally improve the performance of the Collection Channel/AWS system. The model is well suited to incorporate the geometrical and or mechanical/electrical changes to simulate and determine if each of these alternatives would meet or exceed the present biological criteria. The changes to the model to reflect how the prototype would perform would be relatively easy for engineering personnel to insert after a brief time of reviewing the Users Manual and following the simple logic flow of the program. A list of the Preliminary Alternatives and methods of modifying the logic to reflect these suggested changes follows:

1. Replace Motors and Gearboxes to increase Pump Output:
   This type of change was easily accomplished by inserting revised pump coefficients from the regressed modified H-Q curve resulting from the higher impeller RPM

2. Replace Motors, Gearboxes and Impellers:
   The changes to the model would be the same as above with a higher producing pump curve.

3. Enlargement of the Discharge Channel
   The dimensions of the pump discharge channel were easily modified.

4. Removal of the Lowest five ladder weirs (155-159), and increasing their Diffuser Capacity.
   This was a more complex modification to the program logic. The removal of the weirs required an addition of spatially varied flow and linear momentum calculations across the former weir pools with diffusers. The increased diffuser area was easily modified.

Conclusion

The John Day North fishway simulation model was successfully developed to reveal the system problems and to provide preliminary alternative design recommendations. From the evaluation study, a recommendation was made to proceed with an alternative cost study and Design Document Report for the John Day North Fishway upgrade. The model will be used and modified as needed to complete the next phase.

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